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Design of a Coliseum & Exposition Building for Peoria, Ill.

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A DESIGN

OF A

COLISEUM AND EXPOSITION BUILDING

FOR

PEORIA, ILLINOIS

BY

CHRISTIAN PETER LAWRENCE ~~WRENCE~~ PETERSEN
uritz

THESIS FOR DEGREE OF BACHELOR OF SCIENCE
IN CIVIL ENGINEERING

COLLEGE OF ENGINEERING
UNIVERSITY OF ILLINOIS
PRESENTED JUNE 1903

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UNIVERSITY OF ILLINOIS

June 1, 1903

This is to certify that the thesis prepared under the supervision of Assistant Professor Ketchum by CHRISTIAN PETER LAURITZ PETERSEN, entitled DESIGN OF A COLISEUM AND EXPOSITION BUILDING FOR PEORIA, ILLINOIS, is approved by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Ira O. Baker

Head of Department of Civil Engineering

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Introduction.

In most cities a large building for the use of expositions, conventions and for other similar purposes is needed. Such a building is usually a one story structure with a large floorspace and seating capacity. In the Chicago and the St Louis Coliseums the framework consists of three hinged arches, which are designed to support the roof and side walls. In the Kansas City Convention Hall simple roof trusses are supported on the masonry walls. From a comparison of these and other similar structures it was found that the three-hinged arch would be the most economical, and it was therefore adopted for this design.

It is the object of this thesis to make a general design of the building and general detail drawings, and an estimate of the cost of the steelframe work.

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General Specifications

The following general specifications have been adopted for the design of the proposed exposition building and coliseum for Peoria, Illinois.

The building is to have a seating capacity of 4000 people and a clear floor space of 15000 square feet. It is to be constructed of steel and brick and is to be fireproof. The framework is to consist of five-hinged arches lonely braced together. The building is to have a rectangular form. The general dimensions are to be as follows: Length = 304'-3", Width = 158'-3" and height over center line of bottom pins = 102'-0".

Fowlers specifications for steel roofs and buildings revised as noted are to govern the design of the steel work.

Changes and additions to the specifications are as follows:-

§14, page 2. Changed to read:

"Columns ----- $15000 - 55\frac{l}{r}$,

where l = length in inches center to center of connections, r = least radius of gyration"

§16, page 3. Instead of: $S = \frac{Mn}{I} + \frac{P}{A}$

write: $S = \frac{My}{I \pm \frac{Pl^2}{10E}} + \frac{P}{A}$, where

M = moment of transverse force in inch pounds

y = greatest distance from neutral axis to extreme fibre in inches.

P = stress in member in pounds.

l = length of member in inches.

E = 28 000 000.

§ 57, page 6, will be made to read as follows:
"In no compression member shall the ratio of the length to the least radius of gyration exceed 150"

In the design of the pins "Cooper's Specifications for Highway Bridges" will govern.

General Descriptions:

The floor plan and facade of the building shown in Plate 5 gives the dimensions of the audience hall and arrangement of rooms and doors at the entrance. For sizes and purpose of these rooms see Plate 5.

The front of the building is flanked by two towers three stories in height, while the building over the entrance is two stories in height. The second floor over the entrance contains only two rooms, one at each side of entrance; these rooms may serve for any purpose desired. The second and third floors in the towers have the same arrangement of rooms as the first floor. In the top of each tower is a room with stairs to the third floor, and surrounded with a walk 6 feet wide. The entrance annex is covered with a tile roof extending to the end wall of the audience hall. A stone coping built into this wall extends over the top of this roof. Directly over this coping are three large fixed windows, with stained glass so that the direct rays of the sun does not penetrate into the hall. For the main architectural features of the facade see Plate 5.

The outside view of the building as seen from the side is shown on Plate 4.

General Description of the Steel Frame

The Three Hinged Arches.

The arches are circular. The outer or upper chord is vertical to a height of 36'-0" above the centerline of the lower pins, and from that point up the chord is circular. The center of this chord is on the center line between the lower pins, and the radius is 83'-2".

The lower or inner chord is a semicircle with its center 11' 0" above the center of the upper chord, its radius is 67' 3".

The vertical distance from the center of the lower pins to the center pins is 80'-8".

The arches are spaced 25' 0" apart and are divided into panels of 7'-6" long in the circular part.

The arches carry on their top a skylight roof of 45' 0" span, pitch = $\frac{1}{5}$, and are divided into panels of 7'-6" long. This roof is connected to the arches by columns 10' 0" high.

For general outline of the skeleton frame see Plate 1.

Calculation of Loads for Skylight Roof.

The roof is to be covered with "Eakes & Youngs Patent Metallic Skylight without Putty." The glass will be wire rabbled and ¼" thick. On the sides of the roof will be windows which can be swung open horizontally around an axis through the middle of the windows.

Dead Loads:

Weight of truss per square foot	5.5 lbs
" " the glass covering	5.0 "
" " " purlins	4.0 "
Total dead load per square foot of horizontal projection =	14.5 lbs.
Maximum Snow Load:	20.0 "
Minimum " " :	10.0 "

Distance between arches = 25 feet

Width of panel = 7.5 feet.

Dead load per panel = $25 \times 7.5 \times 14.5$ = 2718.75 lbs.

The wind pressure is taken to be 30 pounds horizontally. The inclination of the roof is 23°. The wind pressure normally is computed by the formula: $P_n = P \times \sin\theta^{1.84\cos(\theta-1)}$, where:

P_n = Normal pressure

P = 30 lbs.

$\theta = 23°$

which gives P_n	16.0 lbs.
Wind load per panel joint = $.75 \times 25 \times 16$ =	3000.0 lbs.
" " " of columns = $30 \times 25 \times 10$ =	7500.0 lbs.

The reaction diagrams and truss diagrams for the skylight roof are shown on Plate 1.

The dead load stresses found multiplied by $\frac{20}{14.5} = 1.38$ gives the stresses in the members for

maximum snow loads, one half of these are the stresses for minimum snow loads. The stress sheet is given on Plate I.

The Two Hinged Arches.

Calculation of Loads.

The roof is to be covered with "Ludowici Interlocking Tiles" (I.I. - 9"×16") Catalogue A-° 1/ 1902. The weight of tiles per square foot of horizontal projection is 7.5 lbs. This tiles rests on subpurlins of angles, which are riveted to I beam rafters. Weight of subpurlins per square foot of horizontal projection - 3.0 lbs. The spacing between the subpurlins will be $13\frac{3}{8}$ inch. Under the main purlins will be a coat of plaster "Roeblings System", wire lathing № 20. The wire is 0.035" in diameter, $2\frac{1}{2}$"×4" meshes, galvanized, with stiffening rods of $\frac{1}{4}$" diameter, 1' 6" long and spaced $7\frac{1}{2}$" apart. Under the purlins the lathing is fastened to $\frac{3}{4}$"×$\frac{3}{16}$" bars 15" apart. The plaster will be 1" thick with $\frac{3}{16}$" key. The weight of plaster per square foot of horizontal projection = 10 lbs. The weight of steel including purlins per square foot is assumed to be 13.0 lbs.

Summary of weights.

Tiles per square foot = ----------	7.5 lbs.
Subpurlins and rafters = ----------	3.0 "
Steel frame 13) and Plaster (10) -	23.0 "
Total =	33.5 lbs.

This load acts normally on surface, wherefore the vertical component is taken as the true load on inclined roof. See Plate I. It is found to be = 30 lbs.

Maximum snow load per square foot = 20.0 lbs.
Minimum snow load " " " = 10.0 "

The wind pressure is taken to be 30 pounds horizontally. The normal pressure is found by

the formula: $h_n = P \sin\theta^{1.84} \cos\theta^{2}$ as before. See Plate I for amount.

Calculation of Joint Loads.

Dead Loads.

Joint at lower pin: = 5.5 × 33.5 × 25 = 4610 lbs
Joint between members 1-2: = 5.5 × 4.1 × 33.5 × 25 = 8115 "
" " " : 2-4: = 8.33 × 33.5 × 25 = 6951 "
" " " : 4-6: = 8.33 × 33.5 × 25 = 6951 "
" " " : 6-8: =
4.17 × 33.5 × 25 + 3.75 × 30 × 25 = ----- 6315 "
" " " : 8-9: up to and inclusive
17-19: = 7.5 × 30 × 25 = ---------- 5620 "
Joint between members: 19-21: = 7.5 × 30 × 25 +
8150 (reaction from skylight roof) 13770 "
Joint between 21 23: = 7.5 × 30 × 25 = 5620 "
" " 23-24: = 7.5 × 30 × 25 = ---------- 5620 "
" at center pin: = 3.75 × 30 × 25 = ---- 2815 "

Diagrams for amounts and directions of reactions are shown on Plate I.

Wind Loads.

Diagrams for amounts and directions of reactions are shown on Plate I.

Calculations of Loads due to Balconys and Galleries.

It is assumed the load is 85 lbs pr. square foot of horizontal surface of balcony and gallery, which is a safe load. See Plate I for position of joints.

Joint b: 85 × 25 × 4 = 8500 lbs.
" c: 85 × 25 × 6 = ---------- 12800 "
" a: 85 × 25 × 13 = ---------- 28000 "
" d: 85 × 25 × 12 = ---------- 25500 "

The reaction diagrams are shown on Plate I.

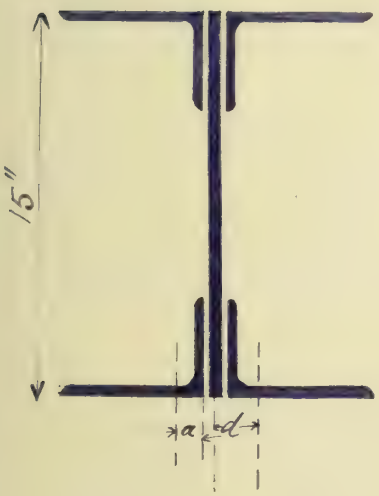
The reactions from dead loads, wind loads, and concentrated loads were transferred to Plate 2, where the stress diagrams were marked out, and a stress sheet compiled. The stresses due to maximum and minimum snow loads were found by multiplying the dead load stresses by the ratio of dead load to snow load. The three last columns on the stress sheet shows the combination of loads for which the members were designed.

Design of Members.

The Three Hinged Arch

Member 1–X

Maximum Stress = + 190350 lbs.



Section:

4 Ls 6 × 4 × $\frac{3}{8}$" Area 14.44 sq. inches
1 web Plate 15" × $\frac{3}{8}$" " = 5.62 " "
Total Area = 20.06 " "

$I = I_{Ls} + A_{Ls} d^2 + I_{pl.}$

$= 4 \times 13.47 + 14.44 \times 2.13^2 + 0.065 = 119.46$

$d = 1.94 + \frac{3}{16} = 2.13$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{119.46}{20.06}} = \sqrt{5.98} = 2.73$

$l = 11'-8" = 140"; \quad \frac{l}{r} = \frac{140}{2.73} = 51.4$

Area required $= \frac{S}{15000 - 55\frac{l}{r}} = \frac{190350}{15000 - 55 \times 51.4} = 15.7$ sq. inches

Since the length of the members in the arch are very short the stress due to their own weight will be so small that it needs not be taken into account.

Member 3–X.

Maximum Stress = + 218900 lbs.

Section: 4 Ls 6 × 4 × $\frac{3}{8}$ — Area = 14.44 sq. inches
1 web Plate 15" × $\frac{3}{8}$ " 5.62 " "
Total Area = 20.06 " "

$l = 8'-4" = 100"$ $\quad \frac{l}{r} = \frac{100}{2.73} = 36.3.$

$r = 2.73$

Area required $= \frac{S}{15000 - 55\frac{l}{r}} = \frac{218900}{15000 - 55 \times 36.3} = 16.85$ sq. inch.

Member 5-X.

Maximum Stress = +208750 lbs.

Section: 4 Ls $6 \times 4 \times \frac{3}{8}$ Area = 14.44 sq. inches

1 web Plate $15 \times \frac{3}{8}$ " " 5.62 " "

Total Area = 20.06 " "

$l = 8'-6" = 102"$ $\quad \frac{l}{r} = \frac{102}{2.73} = 37.4$ Area required

$= \frac{S}{15000 - 55\frac{l}{r}} = \frac{208750}{15000 - 55 \times 37.4} = 16.85$ sq. inches.

Member 7-X.

Maximum Stress = +191700 lbs.

Section: 4 Ls $6 \times 4 \times \frac{3}{8}$ Area = 14.44 sq. inches

1 web plate $15" \times \frac{3}{8}$ " = 5.62 " "

Total Area = 20.06 " "

$l = 14'-10" = 178"$ $\quad \frac{l}{r} = \frac{178}{2.73} = 65.4$ Area required

$= \frac{S}{15000 - 55\frac{l}{r}} = \frac{191700}{15000 - 55 \times 65.4} = 16.8$ sq. inches

Member 10-X.

Maximum Stress = +225750 lbs.

Section: 4 Ls $6 \times 4 \times \frac{3}{8}$ Area = 14.44 sq. inches

1 web Plate $15" \times \frac{3}{8}$" " = 5.62 " "

Total Area = 20.06 " "

$l = 13'-9" = 165"$ $\quad \frac{l}{r} = \frac{165}{2.73} = 60.5$

$r = 2.73$ Area required -

$= \frac{S}{15000 - 55\frac{l}{r}} = \frac{225750}{15000 - 55 \times 60.5} = 20.0$ sq. inches.

Member 12-X.

Maximum Stress = +217850 lbs.

Section: 4 Ls $6 \times 4 \times \frac{3}{8}$ Area = 14.44 sq inches

1 web Plate $15" \times \frac{3}{8}$" " = 5.62 " "

Total Area = 20.06 " "

$l = 10' 11" = 131"$ $\quad \frac{l}{r} = \frac{131}{2.73} = 48.1$ Area required

$r = 2.73$

$= \frac{S}{15000 - 55\frac{l}{r}} = \frac{217850}{15000 - 55 \times 48.1} = 17.6$ square inches

Member 14-X.

Maximum Stress = + 201200 lbs.

Section: $4\ \angle s\ 5 \times 4 \times \frac{3}{8}$ Area = 12.92 sq. inches

1 web Plate $15 \times \frac{3}{8}$ " = 5.62 " "

Total Area = 18.54 " "

$I = I_{\angle s} + A_{\angle s} d^2 + I_{pl} \quad d = 1.53 + \frac{3}{16} = 1.72$

$= 8.14 \times 4 + 3.23 \times 4 \times 1.72^2 + 0.065$

$= 70.765$

$\frac{bd^3}{12} = \frac{3^3 \times 15}{8^3 \times 12} = 0.065$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{70.765}{18.54}} = \sqrt{3.82} = 1.66$

$l = 9' - 6" = 114"$

$\frac{l}{r} = \frac{114}{1.66} = 68.8$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{201200}{15000 - 55 \times 68.8} = 17.9$ sq. inches.

Member 16-X.

Maximum Stress = + 177050 lbs.

Section: $4\ \angle s\ 5 \times 4 \times \frac{3}{8}$ Area = 12.92 sq. inches

1 web Plate: $15 \times \frac{3}{8}$ " = 5.62 " "

Total Area = 18.54 " "

$l = 8' - 8" = 104"$

$r = 1.66 \quad \frac{l}{r} = \frac{104}{1.66} = 62.6$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{177050}{15000 - 55 \times 62.6} = 15.4$ sq. inches.

Member 18-X.

Maximum Stress = + 144350 lbs.

Section: $4\ \angle s\ 5 \times 3 \times \frac{3}{8}$ Area = 11.44 sq. inches

1 web Plate $15 \times \frac{3}{8}$ " = 5.62 " "

Total Area = 17.06 " "

$d = 1.70 + \frac{3}{16} = 1.887$

$I = I_{Ls} + A_{Ls}d^2 + I_{pl} = 4 \times 7.37 + 4 \times 2.86 \times 1.887^2 + 0.065$
$= 29.41 + 40.8 + 0.065 = 70.27$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{70.27}{17.06}} = \sqrt{4.12} = 2.03$

$l = 8'-1" = 97"$ $\quad \frac{l}{r} = \frac{97}{2.03} = 47.8$ $\quad$ Area required =

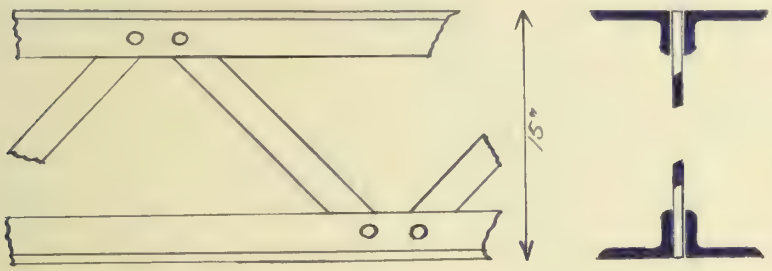
$\frac{S}{15000 - 55\frac{l}{r}} = \frac{144350}{15000 - 55 \times 47.8} = 11.7$ sq. inches.

Member 20 - X.

Maximum Stress = + 105500 lbs.

Section: 4 Ls $5 \times 4 \times \frac{3}{8}$ spaced 15" apart and laced.

Area = 12.94 sq. inches.



$l = 7'-10" = 94"$

$r = 2.34$

$\frac{l}{r} = \frac{94}{2.34} = 39.3$

Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{105500}{15000 - 55 \times 39.3} = 8.22$ sq. inches.

Lacing bars will be: $2" \times \frac{3}{8}"$

Member 22 - X.

Maximum Stress = 78550 lbs.

Section: 4 Ls $5 \times 3 \times \frac{5}{16}$ Area = 9.6 sq. inches

$l = 7'-7" = 91"$

$r = 2.49$ $\quad \frac{l}{r} = \frac{91}{2.49} = 36.6$ $\quad$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{78550}{15000 - 55 \times 36.6} = 6.1$ sq. inches

Lacing bars: $2" \times \frac{3}{8}"$

Member 24 - X

Maximum Stress = + 56200 lbs.

Section: 4 Ls $4 \times 3 \times \frac{5}{16}$ Area = 8.36 sq. inches

$l = 8'-0" = 96"$ $\quad \frac{l}{r} = \frac{96}{1.93} = 49.6$

$r = 1.93$

Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{36200}{15000 - 55 \times 49.6} = 4.6$ square inches.

Lacing bars: $2" \times \frac{3}{8}"$.

Member 1 - Y.

Maximum Stress = + 65950 lbs.

Section: 4 Ls $4 \times 3 \times \frac{5}{16}$ Area = 8.36 sq. inches

$l = 11'-8" = 140"$

$r = 1.93$ $\frac{l}{r} = \frac{140}{1.93} = 72.6$, Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{65950}{15000 - 55 \times 72.6} = 6.0$ square inches

Lacing bars: $2" \times \frac{3}{8}"$

Member 2 - Y.

Maximum Stress = + 62200 lbs.

Section: 4 Ls $4 \times 3 \times \frac{5}{16}$ Area = 8.36 sq. inches

$l = 8'-4" = 100"$

$r = 1.93$ $\frac{l}{r} = \frac{100}{1.93} = 52$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{62200}{15000 - 55 \times 52} = 5.1$ square inches

Lacing bars: $2" \times \frac{3}{8}"$

Members 4-Y & 6-Y

Maximum Stresses: (4-Y) = + 49900 & - 39650 lbs.

(6 Y) + 54700 & - 51550 "

The section will be the same as for member 2-Y.

Member 8-Y.

Maximum Stress = - 110900 lbs.

Section: 4 Ls $5 \times 3 \times \frac{3}{8}$ Area = 11.44 sq. inches

Deducted for $\frac{3}{4}"$ rivets $\frac{7}{8}"$ hole " = 2.4 " "

Net area of section = 9.04 " "

Area required = $\frac{S}{s} = \frac{110900}{15000} = 7.4$ sq. inches

Lacing bars: $2" \times \frac{3}{8}"$

Member 9-Y.

Maximum Stress = −112800 lbs.
Section: 4 Ls 5×3×$\frac{3}{8}$ Area = 11.44 sq. inches
Deducted for 4 $\frac{3}{4}$" rivets " = 2.40 " "
Net area of section = ... 9.04 " "
Area required = $\frac{S}{s} = \frac{112800}{15000} = 7.57$ square inches
Lacing bars: 2" × $\frac{3}{8}$".

Member 11-Y.

Maximum Stresses = +69900 and −133800 lbs.
Section: 4 Ls 5×3×$\frac{3}{8}$ Area = 11.44 sq. inches
Deducted for 4−$\frac{3}{4}$" rivets " = 2.40 " "
Net area of section = ... 9.04 " "
l = 10'−2" = 122"
r = 2.47 $\frac{l}{r} = \frac{122}{2.47} = 49.4$ Area required =
$\frac{S}{15000 - 55\frac{l}{r}} = \frac{69900}{15000 - 55 \times 49.4} = 5.7$ square inches

Area required = $\frac{S}{s} = \frac{133800}{15000} = 8.94$ square inches
Lacing bars: 2" × $\frac{3}{8}$"

Member 13-Y.

Maximum Stresses = +90800 and −134150 lbs.
Section: 4 Ls 5×3×$\frac{3}{8}$ Area = 11.44 sq. inches
Deducted for 4−$\frac{3}{4}$" rivets " = 2.40 " "
Net area of section = ... 9.04 " "
l = 9'−3" = 111"
r = 2.49 $\frac{l}{r} = \frac{111}{2.49} = 44.6$ Area required =
$\frac{S}{15000 - 55\frac{l}{r}} = \frac{90800}{15000 - 55 \times 44.6} = 7.23$ square inches

Area required = $\frac{S}{s} = \frac{134150}{15000} = 8.96$ sq. inches
Lacing bars: 2" × $\frac{3}{8}$"

Member 15-Y.

Maximum Stresses = +94000 and −117850 lbs.
Section: 4 Ls 5×3×$\frac{3}{8}$ Area = 11.44 sq. inches
Deducted for 4−$\frac{3}{4}$" rivets " = 2.40 " "
Net area of section = ... 9.04 " "

$l = 8'-8" = 104"$

$r = 2.49$ $\quad \frac{l}{r} = \frac{104}{2.49} = 41.6$; Area required. =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{94000}{15000 - 55 \times 41.6} = 7.4$ square inches

Area required $= \frac{S}{s} = \frac{117850}{15000} = 7.8$ square inches

Lacing bars: $2" \times \frac{3}{8}"$

Member 17 - Y.

Maximum Stress = +89500 and -90750 lbs.

Section: 4 L's $4 \times 3 \times \frac{3}{8}$ Area = 9.92 sq. inches

Deducted for 4 - $\frac{3}{4}$" rivets " = 2.40 " "

Net area of section = ... 7.52 " "

$l = 8'-2" = 98"$

$r = 1.93$ $\quad \frac{l}{r} = \frac{98}{1.93} = 50.4$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{89500}{15000 - 55 \times 50.4} = 7.33$ square inches

Area required $= \frac{S}{s} = \frac{90750}{15000} = 6.5$ square inches

Lacing bars: $2" \times \frac{3}{8}"$

Member 19 - Y

Maximum Stress = +77700 lbs.

Section: 4 L's $4 \times 3 \times \frac{3}{8}$ Area = 9.92 sq. inches

$l = 7'-10" = 94"$

$r = 1.95$ $\quad \frac{l}{r} = \frac{94}{1.95} = 48$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{77700}{15000 - 55 \times 48} = 6.3$ square inches

Lacing bars: $2" \times \frac{3}{8}"$

Member 21 - A

Maximum Stress = +64450 lbs

Section: 4 L's $4 \times 3 \times \frac{5}{16}$ Area = 8.36 sq. inches

$l = 7'-9" = 93"$

$r = 1.93$ $\quad \frac{l}{r} = \frac{93}{1.93} = 48.2$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{64450}{15000 - 55 \times 48.2} = 5.3$ square inches

Lacing bars: $2" \times \frac{3}{8}"$.

Member 23-A

Maximum Stress = + 47900 lbs.

Section: 4 L's $3\frac{1}{2} \times 3 \times \frac{5}{16}$ Area = 7.74 sq. inches

$l = 7'-7" = 91"$

$r = 1.66$ $\quad \frac{l}{r} = \frac{91}{1.66} = 55$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{47900}{15000 - 55 \times 55} = 41$ square inches

Lacing bars: $2" \times \frac{3}{8}"$

Member 24-A

Maximum Stress = + 49350 lbs.

The section will be the same as for member 23-A.

Web Members.

Member 1-2.

Maximum Stress = − 22350 lbs.

Area required = $\frac{S}{S} = \frac{22350}{15000} = 1.5$ square inches

Add for 2 − $\frac{3}{4}$" rivets area = 1.2 " "

Gross area required = − − − 2.7 " "

Section to be used: 2 L's $3 \times 2 \times \frac{5}{16}$ area = 2.93 sq. inches

Member 2-3.

Maximum Stress = − 53900 lbs.

Area required = $\frac{S}{S} = \frac{53900}{15000} = 3.6$ square inches

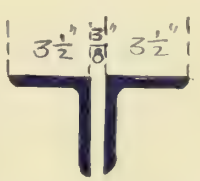
Add for 2 − $\frac{3}{4}$" rivets area = 1.2 " "

Gross area required = − − − − − − 4.8 " "

Section: 2 L's $5 \times 3 \times \frac{5}{16}$ − area = 4.8 " "

Member 3-4.

Maximum Stress = + 37050 lbs



$l = 8'-4" = 100"$

$r = 1.26$ $\quad \frac{l}{r} = \frac{100}{1.26} = 79.5$

Section: 2 L's $4 \times 3\frac{1}{2} \times \frac{5}{16}$ Area = 4.49 sq. inches

Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{37050}{15000 - 55 \times 79.5} = 3.5$ square inches.

Member 4-5

Maximum Stress = − 17325 lbs

Area required = $\frac{S}{s} = \frac{17325}{15000} = 1.15$ square inches

Add for 2 − $\frac{3}{4}$" rivets Area 1.20 " "

Gross area required = 2.35 " "

Section: 2 Ls $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$" area = 2.38 " "

Member 5-6

Maximum Stress = + 12500 lbs.

Section: 2 Ls $3 \times 2 \times \frac{1}{4}$" Area = 2.38 square inches

$l = 9'-11" = 119"$ $\frac{l}{r} = \frac{119}{0.89} = 134$ Area required =

$r = 0.89$

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{12500}{15000 - 55 \times 134} = 1.5$ square inches

Member 6-7

Maximum Stress = + 24850 lbs.

Section =: 2 Ls $4 \times 3 \times \frac{5}{16}$ Area = 4.18 square inches

$l = 13'-0" = 156$ $\frac{l}{r} = \frac{156}{1.27} = 122.6$ Area required

$r = 1.27$

$= \frac{S}{15000 - 55\frac{l}{r}} = \frac{24850}{15000 - 55 \times 122.6} = 3.21$ sq. inches

Member 7-8.

Maximum Stress = + 42600 lbs.

Section: 2 Ls $6 \times 4 \times \frac{3}{8}$. Area = 7.22 sq. inches

$l = 16'-2" = 194"$ $\frac{l}{r} = \frac{194}{1.62} = 119.6$ Area required =

$r = 1.62$

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{42600}{15000 - 55 \times 119.6} = 5.07$ square inches

Member 8-9

Maximum Stress = + 21800 lbs.

Section: 2 Ls $4 \times 3 \times \frac{5}{16}$. Area = 4.18 sq. inches

$l = 10'-5" = 125"$ $\frac{l}{r} = \frac{125}{1.27} = 98.6$ Area required =

$r = 1.27$

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{21800}{15000 - 55 \times 98.6} = 2.3$ square inches

Member 9-10

Maximum Stress = +33885 lbs.

Section: 2 Ls 6 × 3½ × $\frac{3}{8}$. Area = 6.84 sq. inches

l = 16'-3" = 195"

r = 1.39 $\quad \frac{l}{r} = \frac{195}{1.39} = 123$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{33885}{15000 - 55 \times 123} = 4.12 \text{ sq. inches}$$

Member 10-11

Maximum Stress = +30450 lbs

Section: 2 Ls 4 × 3 × $\frac{5}{16}$. Area = 4.18 sq. inches

l = 8'-10" = 106"

r = 1.27 $\quad \frac{l}{r} = \frac{106}{1.27} = 83.6$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{30450}{15000 - 55 \times 836} = 2.93 \text{ square inches}$$

Member 11-12.

Maximum Stress = +27400 lbs

Section: 2 Ls 3½ × 3 × $\frac{5}{16}$. Area = 3.87 sq. inches

l = 11'-6" = 138

r = 1.1 $\quad \frac{l}{r} = \frac{138}{1.1} = 125.3$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{27400}{15000 - 55 \times 125.3} = 3.4 \text{ square inches}$$

Member 12-13

Maximum Stress = +11660 and -14000 lbs.

Section: 2 Ls 2½ × 2½ × ¼. Area = 2.38 sq. inches

Deduct for 2 - ¾" rivets area = 1.2 " "

Net area of section = ------- 1.18 " "

l = 8'-4" = 100"

r = 0.77 $\quad \frac{l}{r} = \frac{100}{0.77} = 130$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{11660}{15000 - 55 \times 130} = 1.48 \text{ square inches}$$

Area required = $\frac{S}{S} = \frac{14000}{15000} = 0.94$ square inches.

Member 13-14.

Maximum Stress = +34900 lbs.

Section: 2 Ls 3½ × 3 × $\frac{5}{16}$ Area = 3.87 sq. inches

l = 9'-0" = 108" $\quad r$ = 1.1

$\frac{l}{r} = \frac{108}{1.1} = 99.4$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{34900}{15000 - 55 \times 99.4} = 3.66$ sq uare inches

Member 14-15.

Maximum Stress = -19400 lbs.

Area required = $\frac{S}{s} = \frac{19400}{15000} = 1.3$ square inches

Add for 2 - $\frac{3}{4}$" rivets area = 1.2 " "

Gross area required = 2.5 " "

Section: 2 Ls 3 × 2 × $\frac{5}{16}$. Area 2.93 " "

Member 15-16.

Maximum Stress = +38200 lbs.

Section: 2 Ls 5 × 3 × $\frac{5}{16}$. Area = 4.8 sq. inches

$l = 7'-6" = 90$

$r = 1.22$ $\frac{l}{r} = \frac{90}{1.22} = 73.6$. Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{38200}{15000 - 55 \times 73.6} = 3.5$ sq. inches

Member 16-17.

Maximum Stress = -26000 lbs

Area required = $\frac{S}{s} = \frac{26000}{15000} = 1.75$ square inches

Add for 2 - $\frac{3}{4}$" rivets area = 1.20 " "

Gross area required = 2.95 " "

Section: 2 Ls 3 × 2$\frac{1}{2}$ × $\frac{5}{16}$ area = 3.22 " "

Member 17-18

Maximum Stress = +41450 lbs.

Section: 2 Ls 4 × 3 × $\frac{5}{16}$ Area = 4.18 sq. inches

$l = 6'-5" = 77"$

$r = 1.27$ $\frac{l}{r} = \frac{77}{1.27} = 60.6$, Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{41450}{15000 - 55 \times 60.6} = 3.55$ sq. inches

Member 18-19.

Maximum Stress = -34300 lbs.

Area required = $\frac{S}{s} = \frac{34300}{15000} = 2.3$ square inches

Add for 2 - $\frac{3}{4}$" rivets area = 1.2 " "

Gross area required = 3.5 " "

Section: 2 ls $3\frac{1}{2} \times 3 \times \frac{5}{16}$ Area = 3.87 sq. inches

Member 19-20

Maximum Stress = + 41350 lbs

Section: 2 ls $4 \times 3 \times \frac{5}{16}$ Area = 4.18 sq inches

$l = 5'-9" = 69"$

$r = 1.27$ $\quad \frac{l}{r} = \frac{69}{1.27} = 54.4$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{41350}{15000 - 55 \times 54.4} = 3.44 \text{ sq. inches}$$

Member 20-21

Maximum Stress = + 25000 and - 27000 lbs

Section: 2 ls $3\frac{1}{2} \times 3 \times \frac{5}{16}$ Area = 3.87 sq inches

Deduct for 2 - $\frac{3}{4}$" rivets area = 1.20 " "

Net area required = ... 2.67 " "

$l = 8'-4" = 100$

$r = 1.1$ $\quad \frac{l}{r} = \frac{100}{1.1} = 91$, Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{25000}{15000 - 55 \times 91} = 2.5 \text{ square inches}$$

$$\text{Area required} = \frac{S}{s} = \frac{27000}{15000} = 1.8 \text{ square inches}$$

Member 21-22.

Maximum Stress = + 27350 lbs.

Section: 2 ls $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ Area = 2.88 sq inches

$l = 5'-4" = 64"$

$r = 1.09$ $\quad \frac{l}{r} = \frac{64}{1.09} = 58.6$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{27350}{15000 - 55 \times 58.6} = 2.32 \text{ sq. inches}$$

Member 22-23.

Maximum Stress = + 27300 and - 29350 lbs.

Section: 2 ls $3\frac{1}{2} \times 3 \times \frac{5}{16}$, Area = 3.87 sq. inches

Deduct for 2 - $\frac{3}{4}$" rivets area = 1.20 " "

Net area required - 2.67 " "

$l = 8'-7" = 103$

$r = 1.1$ $\quad \frac{l}{r} = \frac{103}{1.1} = 93.5$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{27300}{15000 - 55 \times 93.5} = 2.77 \text{ sq. inches}$$

$$\text{Area required} = \frac{S}{s} = \frac{29350}{15000} = 1.95 \text{ " "}$$

Member 23-24

Maximum Stress = −13800 lbs

Area required = $\frac{S}{s} = \frac{13800}{15000} = 0.92$ square inches

Add for 2 − $\frac{3}{4}$" rivets area = 1.20 " "

Gross area required = 2.12 " "

Section: 2 Ls $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$, area 2.38 " "

Design of Members in Skylight Roof.

Member 25-Y

Maximum Stress = +20400 lbs.

Section: 4 Ls $3 \times 2\frac{1}{2} \times \frac{1}{4}$ Area = 5.26 sq. inches

The angles are laced, depth of columns = 12"

$l = 10'-0" = 120"$

$r = 1.45$ $\frac{l}{r} = \frac{120}{1.45} = 83$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{20400}{15000 - 55 \times 83} = 1.96$ square inches

Lacing bars: $1\frac{3}{4}" \times \frac{5}{16}"$

Member 25-A

Maximum Stress = +9500 lbs.

Section = 2 Ls $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ Area = 2.88 sq. inches

$l = 12'-6" = 150$

$r = 1.12$ $\frac{l}{r} = \frac{150}{1.12} = 134$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{9500}{15000 - 55 \times 134} = 1.24$ square inches

Members 26-Y, 28-Y & 30-Y

Maximum Stress = +44300 lbs

Section: 2 Ls $4 \times 3\frac{1}{2} \times \frac{5}{16}$ Area = 4.49 sq. inches

$l = 8'-1" = 97"$

$r = 1.26$ $\frac{l}{r} = \frac{97}{1.26} = 77$ Area required =

$\frac{S}{15000 - 55\frac{l}{r}} = \frac{44300}{15000 - 55 \times 77} = 4.11$ sq. inches

Members 25-26, 27-A & 29-A

Maximum Stress = −40200 lbs.

Area required = $\frac{S}{s} = \frac{40200}{15000} = 2.70$ sq. inches

Add for 2 − $\frac{3}{4}$" rivets area = 1.20 " "

Gross area required = 3.90 " "

Section: 2 Ls $4 \times 3 \times \frac{5}{16}$ Area = 4.18 " "

Member 26 - 27

Maximum Stress = +7700 lbs.

Section: 2 Ls 2 × 2 × ¼ Area = 1.88 square inches

l - 3'-0" = 36

r = 0.61 $\quad \frac{l}{r} = \frac{36}{0.61} = 59$ Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{7700}{15000 - 55 \times 59} = 0.66 \text{ square inches}$$

Member 27 - 28

Maximum Stress = +13700 lbs.

Section: 2 Ls 2½ × 2½ × ¼ Area = 2.38 sq inches

l - 8'-1" = 97

r = 0.77 $\quad \frac{l}{r} = \frac{97}{0.77} = 126$ Area required -

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{13700}{15000 - 55 \times 126} = 1.70 \text{ square inches}$$

Member 28-29

Maximum Stress = +3800 and -4800 lbs.

Section: 2 Ls 2 × 2 × ¼ Area = 1.88 sq. inches

l = 6'-0" = 72

r = 0.61 $\quad \frac{l}{r} = \frac{72}{0.61} = 118$

Deduct for 2 - ¾" rivets area = 1.20 " "

Net area required = ------- 0.68 " "

Area required for compression =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{3800}{15000 - 55 \times 118} = 0.45 \text{ sq. inches}$$

$$\text{Tension area required} = \frac{S}{S} = \frac{4800}{15000} = 0.32 \text{ sq. inches}$$

Member 29 - 30

Maximum Stress = +13900 lbs.

Section: 2 Ls 2½ × 2½ × ¼ Area = 2.38 sq. inches

l = 9'-7" = 115

r = 0.77 $\quad \frac{l}{r} = \frac{115}{0.77} = 149.3$; Area required =

$$\frac{S}{15000 - 55\frac{l}{r}} = \frac{13900}{15000 - 55 \times 149.3} = 2.06 \text{ sq. inches}$$

Member 30 31.

Maximum Stress = 12900 lbs

Area required = $\frac{5}{5} \cdot \frac{12900}{15000}$ = 0.86 square inches

Add for 2 - $\frac{3}{4}$" rivets area = 1.20 " "

Gross area required = ------ 2.06 " "

Section: 1 L^s 3 × 2 × $\frac{5}{16}$ Area = 2.93 " "

Design of Purlins on Main Roof.

A purlin will be laid over each joint of the arch. They will extend over the last two arches into the end walls where they will rest on bed plates built into the wall.

Calculation of Loads.

Dead load = 7'-6" × 25'-0" × (3.0 + 7.5 + 10.0)

= 7.5 × 25 × 20.5 = 3841 lbs

Max. Snow Load = 7.5 × 25 × 20 = 3760 "

Min. " " = 7.5 × 25 × 10 = 1880 "

Wind Load for average inclination of roof

= 7.5 × 25 × 20.4 = 3820 "

Dead Load + Max. Snow Load = 3841 + 3760 = 7601 "

Dead Load + Min. Snow Load + Wind Load

= 3841 + 1880 + 3820 = 9541 "

which load will be used in designing.

Allowable unit stress = 15000 lbs

Section: 10" I Beams @ 25 lbs.

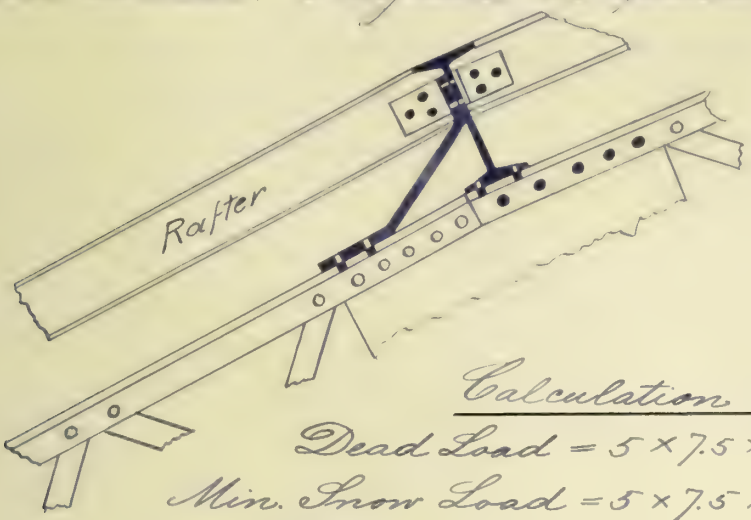
$S = \frac{My}{I} = \frac{\frac{1}{8} W l y}{I} = \frac{(25 \times 25 + 9541) 25 \times 12 \times 5}{8 \times 122.1} =$

$\frac{15210000}{976.8}$ = 15600 lbs. extreme fibre stress.

Although this is a little greater than the allowable unit stress the section will be used, since the actual unit stress found was calculated for beams supported at the ends, while they in reality are continuous girders.

Design of Rafters.

The rafters will consists of I beams and fastened to the purlins as shown on Plate 3 and on sketch. They will be spaced 5'-0" apart.



Calculation of Loads.

Dead Load = $5 \times 7.5 \times 10.5$ = 394 lbs.
Min. Snow Load = $5 \times 7.5 \times 10$ = 375 "
Wind Load = $5 \times 7.5 \times 20.4$ = 765 "
Total load used for design = ---------- 1534 "

Section: 4" [@ 5.25 lbs.

$$S = \frac{My}{I} = \frac{Wly}{8I} = \frac{(1534+52.5)\times 10\times 12\times 2}{8\times 3.8} =$$

$$\frac{1586.5\times 240}{8\times 3.8} = \frac{15865\times 3}{3.8} = 12500 \text{ lbs. extreme fibre}$$

unit stress. Allowable stress = 15000 lbs.

The rafters will extend from the arch over to the side walls, where they will be connected to a 10" I beam @ 25 lbs, which is laid in the masonry. The lowest two rafters will, owing to thier greater length, be made 5" [@ 6.5 lbs also will they have to take a greater wind load, since they are more inclined.

Design of Purlins at Foot of Skylight Roof.

Calculation of Loads.

Dead Load = $3 \times 25 \times 20.5$ = 1537.5 lbs
Min. Snow Load = $3 \times 25 \times 10$ = 750 "
Wind Load = $3 \times 25 \times 20.4$ = 1530 "
Total Load ---------- = 3817 "

Section: 10" [@ 15 lbs.
l = 25 feet.

$S = \frac{My}{I} = \frac{Wly}{8I} = \frac{[(25 \times 15) + 3817] \times 25 \times 12 \times 5}{8 \times 66.9} = 11730$ lbs.

Allowable unit stress = 15000 lbs.

Design of Sub-purlins.

Calculation of Loads

Dead Load = $13\frac{3}{8} \times 5 \times 7.5 = 1.11146 \times 375$ = 420 lbs.

Min. Snow Load = $1.11146 \times 5 \times 10$ = ... 560 "

Wind Load = $1.11146 \times 5 \times 20.4$ = ... 1140 "

Total Load = ... 2120 "

Section: L^s $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{16}$ spaced $13\frac{3}{8}$" apart on the rafters.

$S = \frac{My}{I} = \frac{Wly}{8I} = \frac{[(5 \times 1.97) + 212] \times 3 \times 12 \times \frac{3}{4}}{8 \times 0.11} = 11350$ lbs.

Allowable unit stress = 15000 lbs.

Design of Balcony Girts at Joints b-b

For position of joints see Plates 1 and 2.

Load on Girt = $85 \times 25 \times 4'\text{-}0"$ = 8500 lbs.

Allowance for longitudinal truss bracing = 2000 "

Total Load = ... 10500 "

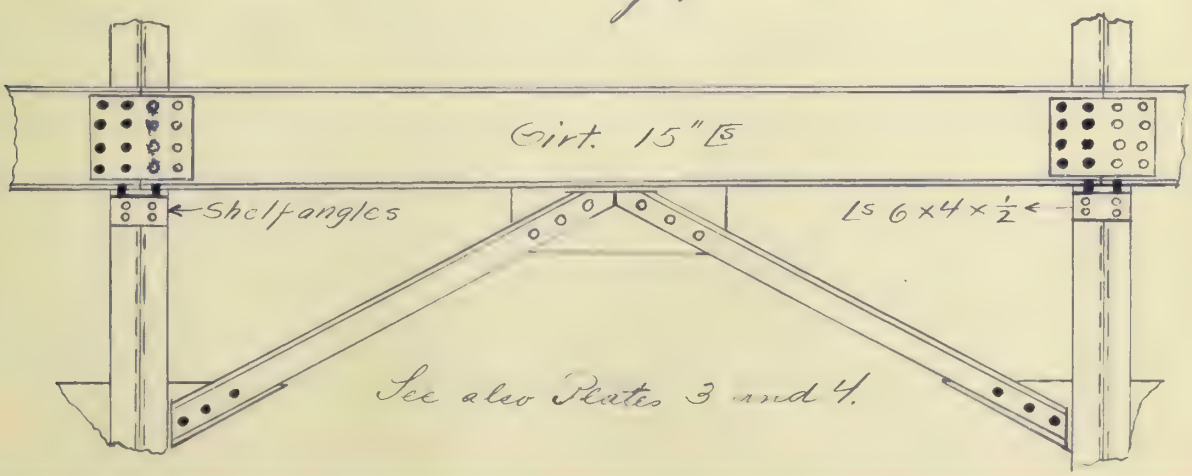
Selected from "Cambria Steel" handbook a 15" [s @ 33 lbs safe load = 17780 lbs for extreme fibre stress of 16000 lbs.

Extreme fibre stress for actual loading would be:

$f = \frac{My}{I} = \frac{Wly}{12I} = \frac{[(33 \times 25) + 10500] \times 12 \times 25 \times 7.5}{12 \times 312.6} = 6830$ lbs.

Allowable unit stress = ... 15000 "

To secure perfect safety the girt will be stiffened by two knee braces each consisting of 2 L^s 3"×3"×$\frac{1}{4}$".



See also Plates 3 and 4.

Gallery Girt at joints c c

Load on girt = 85 × 25 × 6.5 = 12800 lbs.
Allowance for longitudinal bracing = 2200 "
Total load = 15000 "
Selected from "Cambria Steel" handbook = 15" L's @ 40 lbs.

$$f = \frac{My}{I} = \frac{Wey}{12I} = \frac{[(40 \times 25) + 15000] \times 12 \times 25 \times 7.5}{12 \times 347.5} = 8650 \text{ lbs.}$$

Allowable Stress = 15000 lbs. For further safety two kneebraces each consisting of 2 L's 3 × 3 × ¼ will be used.

Balcony Girt at joints a-a

Load on girt 85 × 25 × 13.2 = 28000 lbs.
Selected from "Cambria Steel" handbook 15" L's @ 55 lbs.

$$f = \frac{My}{I} = \frac{Wey}{12I} = \frac{[(55 \times 25) + 28000] \times 12 \times 25 \times 7.5}{12 \times 430.2} = 12{,}00 \text{ lbs.}$$

Allowable stress = 15000 lbs. For additional safety two kneebraces @ 2 L's 6 × 4 × $\frac{5}{16}$ will be used.

Gallery Girt at joints d d.

Load on girt 85 × 25 × 12 = 25500 lbs.
Same sections as for girt at joints a-a will be used.
For arrangement of girts see Plates 3 and 4 and sketch on page 24.

Balcony joists.

The joists are spaced 5'-0" apart. l = 23'-0"
Load on joists = 23 × 5 × 85 = 9800 lbs.
Section: 10" I beams @ 25 lbs.

$$f = \frac{My}{I} = \frac{Wey}{8I} = \frac{[(25 \times 23) + 9800] \times 23 \times 12 \times 5}{8 \times 122.1} = 14700 \text{ lbs}$$

Allowable unit stress = 15000 lbs. In the wall under the lower end of the joists is laid a 4" I beam @ 15 lbs on which rests the balcony and floor joists. The 4" I beam rests on a bid plate of $\frac{3}{8}$" thickness.
For arrangement of joists see Plates 3 and 4.

Floor Joists for Box Floor.

Joists are spaced 5'0" c. to c. $l = 11'6"$

Load on joists $= 5 \times 11.5 \times 85 = 4880$ lbs

Section: 6" I beams @ 12.25 lbs.

$$f = \frac{My}{I} = \frac{Wly}{8I} = \frac{[(12.25 \times 11.5) + 4880] \times 11.5 \times 12 \times 3}{8 \times 21.8} = 10450"$$

Allowable stress = 15000 lbs. The floor joists rest on a 4" I beam on the inner wall as described under balcony joists.

Gallery Joists.

Joists are spaced 5'0" c. to c. They act both as simple beams and as cantilevers and will be designed for both.

Cantilevers. $l = 6'6" = 6.5$, Section: 10" I @ 25 lbs

Load on cantilever $= 85 \times 5 \times 6.5 = 2760$ lbs

$$f = \frac{My}{I} = \frac{Wly}{2I} = \frac{[(25 \times 6.5) + 2760] \times 12 \times 6.5 \times 5}{2 \times 122.1} = 11610 \text{ lbs.}$$

Beams. Load on beam $= 85 \times 5 \times 13 = 5520$ lbs.

$l = 13'0"$. $f = \frac{My}{I} = \frac{Wly}{8I} = \frac{[(25 \times 6.5) + 5520] \times 12 \times 13 \times 5}{8 \times 122.1} = 4540$ lbs.

Allowable stress = 15000 lbs. Although 10" I beams are much too large for the load used in the investigation, this section will nevertheless be used, since a gallery is liable to sudden concentration of loads.

Truss Bars.

These bars, on which the floors on balcony and gallery are laid, will act both as columns and beams, and will be designed for both. $l = 3'0" = 3.0$

$h = 1'-9" = 1.75$

Beams. Load on bar $= 3 \times 5 \times 85 = 1270$ lbs.

Section: flat steel bars 5" × 1". weight pr. foot = 17 lbs.

$$f = \frac{My}{I} = \frac{Wly}{8I} = \frac{[(17 \times 3) + 1270] \times 12 \times 3 \times 12}{2 \times 8 \times 5} = 7150 \text{ lbs.}$$

Allowable stress = 15000 lbs.

Columns: Load on column = $\frac{1}{2} \times 1321 = 660$ lbs

$l = 1.75 \times 12 = 21$ $\quad \frac{l'}{r} = \frac{21}{0.289} = 72.6$ $\quad$ Area required =

$\frac{S}{15000 - 55\frac{l'}{r}} = \frac{660}{15000 - 55 \times 72.6} = 1.01$ square inches,

Actual area = 5.0 square inches.

Purlins on Skylight Roof

A purlin will be laid over each joint as shown on Plate 3.

Calculation of Loads:

Dead Load = $7.5 \times 25 \times 10.5 =$ ---- 1965 lbs.

Maximum Snow Load = $7.6 \times 25 \times 20 =$ 3800 "

Minimum " " = $\frac{1}{2} \times 3800 =$ --- 1900 "

Wind Load = $7.6 \times 25 \times 16 =$ ---------- 3020 "

Dead Load + Maximum Snow = 1965 + 3800 = 5765 lbs.

Dead Load + Minimum Snow + Wind Load =

1965 + 1900 + 3020 = 6885 lbs.

Section: 10" [@ 25 lbs.

$f = \frac{My}{I} = \frac{Wyl}{8I} = \frac{[(25 \times 25) + (6885) \times 25 \times 12 \times 5}{8 \times 91.0} = 15400$ lbs.

Although this is a little greater than the 15000 lbs allowable unit stress, the section will be used, since the actual unit stress found was calculated for beams supported at the ends, while they are really are continuous girders. On the purlins will be laid "Vaile & Youngs Patent Metallic Skylight without Putty." The glass is to be one quarter inch wire rolled.

Girt on Sides of Skylight Columns.

Maximum Load = $5 \times 25 \times 30 = 3750$ lbs.

Section: 9" [@ 13.25 lbs.

$f = \frac{My}{I} = \frac{Wly}{8I} = \frac{[(13.25 \times 25) + 3750] \times 12 \times 25 \times 4.5}{8 \times 47.3} = 14500$ lbs.

Allowable Stress = 15000 lbs.

Window Framing.

The window posts will be 4" x 4" and 5' 0" apart
$l = 7'0"$. Max. Load $= 30 \times 5 \times 7 = 1050$ lbs.
Section: 2 Ls $4 \times 3 \times \frac{3}{8}$

$$f = \frac{My}{I} = \frac{Wly}{8I} = \frac{1050 \times 12 \times 7 \times 2.22}{4 \times 1.92 \times 2} = 12720 \text{ lbs.}$$

Allowable unit stress = 15000 lbs.

Bracing Rods.

The bracing rods will be arranged and have the diameter as given on Fig. 4.

To take the stress due to the horizontal component of the reactions each pair of the lower pins will be connected by two rods located under the main floor. These rods will have loop welded eyes and turnbuckles.

Maximum horizontal component H = 50000 lbs
Stress on one rod = — 25000 lbs.

$$\text{Area required} = \frac{P}{S} = \frac{25000}{15000} = 1.66 \text{ square inches}$$

Length of rod center to center = 142' 3" = 142.25 ft.
Range of temperature = 150° Fah. $\alpha = 0.0000065$

$$\text{Stress due to temp.} = \frac{\alpha t E}{A} = \frac{0.0000065 \times 150 \times 28000000}{4 \text{ (2"x2" rod)}} = 6300$$

$$\text{Area required} = \frac{S}{S} = \frac{6300}{15000} = 0.42 \text{ square inches}$$

As the rods will be supported their whole length no stress due to bending will exist.
Total area required = 1.66 + 0.42 = 2.08 sq. inches
The bars will be made 2" square. Area = 4 sq. inches.

Bearing on Pins. The lower pins are 5 inches in diameter see page 29.
Bearing area = 5 x 2 = 10 square inches
Allowable bearing stress = 20000 lbs.

$$\text{Bearing area required} = \frac{31300}{20000} = 1.55 \text{ sq inches.}$$

Design of Pins.

Lower Pin.

Maximum vertical components of reactions 226400 lbs. "Code of Specifications for Highway Bridges" will govern the design of the pins. A 5" pin will be used. The $\frac{3}{8}$" web plate in the arch will not have sufficient bearing and will therefore be reinforced with two $\frac{5}{8}$" plates on each side. Bearing area required = $\frac{226400}{20000} = 11.32$ sq. inches

Actual bearing area = $5 \times \left| \frac{3}{8} + 4 \times \frac{5}{8} \right| = 14.375$ sq. inches

Rivets needed in connecting these plates to web:

Thickness of web plate = $\frac{3}{8}$ = ... 0.375 inches

" " reinf. " = $\frac{5}{8} \therefore 0.625 \times 4 =$ 2.500 "

Total thickness = ... 2.875 "

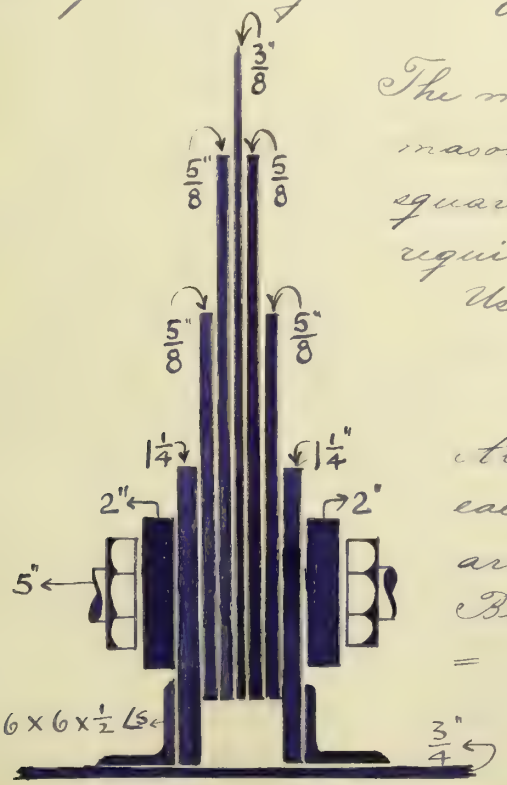
Stress taken by web = $\frac{226400 \times 0.375}{2.875} = 29610$ lbs.

" " " one plate = $\frac{226400 \times 0.625}{2.875} = 49300$ lbs.

$\frac{7}{8}$" rivets will be used for all pin connections, Area of one = 0.6013 square inches.

Rivet area required = $\frac{49300}{10000} = 4.93$ square inches

Number of rivets required = $\frac{4.93}{0.6013} = 8.$



The maximum pressure on the masonry allowed is 300 lbs. per square inch. Area of bed plate required = $\frac{226400}{300} = 755$ sq. inches.

Use plate: 30" x $\frac{3}{4}$ x 2'-6"

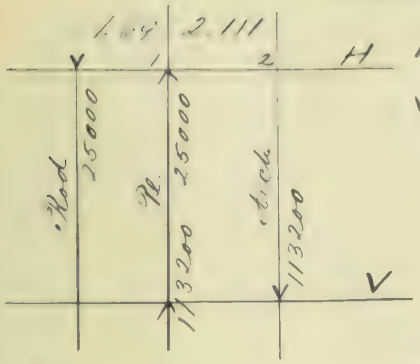
Bearing on Pin.

Allowable unit stress on each square inch bearing area equal 20000 lbs.

Bearing area of arch on pin = $5 \times 2.875 = 14.375$ sq. inches

Unit bearing stress = $\frac{226400}{14.375} = 15800$ lbs.

Bending Moment.



M. at (1) = 25000 × 1.69 = 42250 inch. lbs.
M. at (2) = 113200 × 2.111 = 241000 " "
Max. bending Moment = 241000 " "
Allowable bending stress = 25000 lbs.
From "Cambria Steel" handbook a 5" pin allows a maximum bending moment of 306800 lbs. for a unit stress of 25000 lbs.

Shear.

Maximum shear = 113200 lbs. Area of pin = 19.635 square inches.
Unit shear = $\frac{113200}{19.635}$ = 5800 lbs.
Allowable shear = 10000 lbs.

Dimension of Pin

Packing: $(\frac{3}{16} + \frac{1}{16} + \frac{5}{8} + \frac{11}{16} + \frac{5}{8} + \frac{1}{16} + 1\frac{1}{4} + \frac{1}{16} + 2 + \frac{1}{16})2 = 10"$
Pin: $5" \times 10" \times 12\frac{1}{2}"$

Pedestal.

The pedestal consists of two plates $30" \times 1\frac{1}{4}" \times 1'-6\frac{5}{8}"$ connected to the bed plates by angles $6" \times 6" \times \frac{1}{2}"$.
Stress transmitted through on plate = 113200 lbs.
Allowable bearing stress = 20000 lbs. pr. square inch.
Bearing area needed = $\frac{113200}{20000}$ = 5.7 " "
Actual bearing area = 5 × 1.25 = 6.25 " "

Connection of Angles.

$\frac{7}{8}"$ rivets will be used. Area of one = 0.6013 sq inch
Bearing: bearing area = 1.25 × 0.6013 = 0.725 sq. inches
Shear: thickness of plate = $1\frac{1}{4}"$ = 1.25 inches
" " angles = $\frac{1}{2}"$ = 0.50 "
Total thickness = 1.75 "
Shear taken by plate = $\frac{113200 \times 1.25}{1.75}$ = 81000 lbs.
Shearing area needed = $\frac{81000}{10000}$ = 8.1 square inches.
Number of $\frac{7}{8}"$ rivets required = $\frac{8.1}{0.6013}$ = 14. use 14.

Bearing: Shear on each rivet = $\frac{81000}{17}$ = 4770 lbs.
Bearing area of one rivet = 1.013 × 1.25 = 1.752 sq. inches
for plate. Bearing unit stress = $\frac{4770}{0.752}$ = 6350 lbs.
Unit bearing stress for angles = $\frac{4770}{0.5 \times 0.6613}$ = 15830 lbs
Allowable bearing stress for rivets = 20000 lbs.
For details of lower pins see Plate 3.

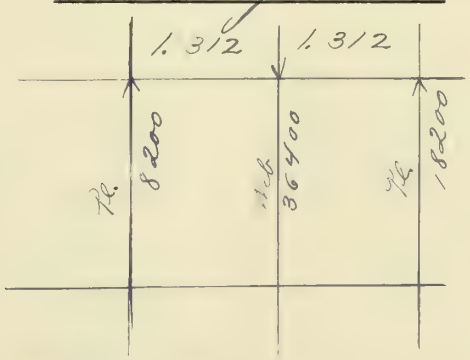
Foundation for Arches.

The foundations will be made of concrete.
Vertical reaction = 226400 lbs. Assuming the bearing power of the soil to be two tons per square foot the base required = $\frac{226400}{4000}$ = 56.6 sq. feet
They will be made 8'0" × 4'0" × 6'0". See Plate 3.

Design of Center Pin.

Maximum Stress at Pin = +50000 lbs.
Pin to be 3 inches in diameter
Bearing area required = $\frac{50000}{20000}$ = 2.5 sq. inches
Actual bearing area =
$(\frac{3}{8} + \frac{1}{2} + \frac{1}{2})3 = 4\frac{1}{8}$ square inches

Bending Stress.



Stress taken by one plate = $\frac{0.5 \times 50000}{0.5 + 0.5 + 0.375}$
= $\frac{25000}{1.375}$ = 18200 lbs.
Maximum bending moment M =
18200 × 1.312 = 27250 lbs. From "Cambria Steel" handbook a 3" pin with 25000 lbs. unit stress allows M. to be 66220 lbs.

Maximum shear = 18200 lbs.

Unit shear = $\frac{18200}{7.0686}$ = 2580 lbs. Allowable shearing unit stress = 10000 lbs.

Dimensions of Pin.

Packing: $\left(\frac{3}{16}+\frac{1}{16}+\frac{1}{2}+\frac{1}{16}+\frac{1}{2}+\frac{1}{16}\right)2$ = 2.75 inches

Size: $3" \times 2\frac{3}{4}" \times 5'-0"$

Rivets required.

$\frac{7}{8}$" rivets will be used. Area of one = 0.6013 sq."

Stress taken by one plate = 18200 lbs.

" " " web = 13600 "

Rivet area required in shear = $\frac{18200}{0.6013}$ = 1.82 sq inches.

Number of rivets required = $\frac{1.82}{0.6013}$ = 3.

Bearing area of web = $\frac{3}{8} \times \frac{7}{8}$ = 0.33 square inches

Bearing stress of one rivet = 12000 × 0.6013 = 12.000 lbs.

Number of rivets required in web for bearing = $\frac{13600}{12000}$ = 2

" " " plate " " = $\frac{18200}{12000}$ = 2.

For details of center pin see Plate 3.

Estimate of Seating Capacity

Balconies: Inside length = 268 ft.

Width of one seat	1.5"	
Number of rows	8	
Total seats on balconies	2859	
Subtracting for four stairs	213 seats	
Total seats — 2859 − 213 =		2646

Boxes: Inside length = 268 ft.

Average width of one box	5"	
Number of rows	2	
Total number of seats		429

Galleries: Inside length = 268 ft.

Width of one seat	.5"	
Number of rows	3.	
Total number of seats		1072

Total seating capacity = 4147 people.

The roofs are suspended from the girders between the arches and the side walls down to the balconies, where exits will be secured through the windows in case of fire.

Floor Space

Inside Length of Floor = 267 feet
Width = 58'-11". Floor space = 15750.29 feet.

Estimate of Weight of Steel Frame

One Arch.

1	2	3	4	5	6	7	8	9
Member.	Kind of Section	No. of Pieces	Cross-Section	Length in Feet.	Weight per Foot.	% Details	Total Weight. Main Members	Details.
1-X	Ls	4	$6\times4\times\frac{3}{8}$	11.67	12.3		575	
	Pl.	1	$15\times\frac{3}{8}$	11.67	19.13	5	225	40
3-X	Ls	4	$6\times4\times\frac{3}{8}$	8.33	12.3		410	
	Pl	1	$15\times\frac{3}{8}$	8.33	19.13	5	159	28
5-X	Ls	4	$6\times4\times\frac{3}{8}$	8.50	12.3		417	
	Pl	1	$15\times\frac{3}{8}$	8.50	19.13	5	162	29
7-X	Ls	4	$6\times4\times\frac{3}{8}$	14.83	12.3		732	
	Pl	1	$15\times\frac{3}{8}$	14.83	19.13	5	284	51
10-X	Ls	4	$6\times4\times\frac{3}{8}$	13.75	12.3		626	
	Pl	1	$15\times\frac{3}{8}$	13.75	19.13	5	263	44
12-X	Ls	4	$6\times4\times\frac{3}{8}$	10.92	12.3		538	
	Pl	1	$15\times\frac{3}{8}$	10.92	19.13	5	209	37
14-X	Ls	4	$5\times4\times\frac{3}{8}$	9.5	11.0		417	
	Pl	1	$15\times\frac{3}{8}$	9.5	19.13	5	182	30
16-X	Ls	4	$5\times4\times\frac{3}{8}$	8.67	11.0		381	
	Pl	1	$15\times\frac{3}{8}$	8.67	19.13	5	165	28
18-X	Ls	4	$5\times3\times\frac{3}{8}$	8.08	9.70		313	
	Pl	1	$15\times\frac{3}{8}$	8.08	19.13	5	155	23
			Total				7213	310

1	2	3	4	5	6	7	8	9
20 - X	Ls	4	5 × 4 × $\frac{3}{8}$	7.83	11.0	40	344	137
22 - X	Ls	4	5 × 3 × $\frac{5}{16}$	7.58	8.2	40	248	99.5
24 - X	Ls	4	4 × 3 × $\frac{5}{16}$	8.0	7.1	40	227	91
1 - Y	Ls	4	4 × 3 × $\frac{5}{16}$	11.67	7.1	40	331	132
2 - Y	Ls	4	4 × 3 × $\frac{5}{16}$	8.33	7.1	40	236	94.5
4 - Y	Ls	4	4 × 3 × $\frac{5}{16}$	8.33	7.1	40	236	94.5
6 - Y	Ls	4	4 × 3 × $\frac{5}{16}$	8.33	7.1	40	236	94.5
8 - Y	Ls	4	5 × 3 × $\frac{3}{8}$	14.75	9.7	40	571	228
9 - Y	Ls	4	5 × 3 × $\frac{3}{8}$	11.75	9.7	40	445	182
11 - Y	Ls	4	5 × 3 × $\frac{3}{8}$	10.17	9.7	40	394	157
13 - Y	Ls	4	5 × 3 × $\frac{3}{8}$	9.25	9.7	40	358	143
15 - Y	Ls	4	5 × 3 × $\frac{3}{8}$	8.67	9.7	40	336	135
17 - Y	Ls	4	4 × 3 × $\frac{3}{8}$	8.17	8.4	40	274	109
19 - Y	Ls	4	4 × 3 × $\frac{3}{8}$	7.83	8.4	40	263	105
21 - A	Ls	4	4 × 3 × $\frac{5}{16}$	7.75	7.1	40	219	88
23 - A	Ls	4	$3\frac{1}{2}$ × 3 × $\frac{5}{16}$	7.58	6.6	40	200	80
24 - A	Ls	4	$3\frac{1}{2}$ × 3 × $\frac{5}{16}$	8.08	6.6	40	214	85.5
1 - 2	Ls	2	3 × 2 × $\frac{5}{16}$	7.75	5.0	2	77	1.5
2 - 3	Ls	2	5 × 3 × $\frac{5}{16}$	11.417	8.2	2	188	4
3 - 4	Ls	2	4 × $3\frac{1}{2}$ × $\frac{5}{16}$	8.33	7.6	2	127	2.5
4 - 5	Ls	2	$2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{1}{4}$	11.75	4.0	2	94	2
5 - 6	Ls	2	3 × 2 × $\frac{1}{4}$	9.917	4.0	2	79	1.5
6 - 7	Ls	2	4 × 3 × $\frac{5}{16}$	13.00	7.1	2	184	3.5
7 - 8	Ls	2	6 × 4 × $\frac{3}{8}$	16.167	12.3	2	398	8
8 - 9	Ls	2	4 × 3 × $\frac{5}{16}$	10.417	7.1	2	148	3
9 - 10	Ls	2	6 × $3\frac{1}{2}$ × $\frac{3}{8}$	16.25	11.6	2	378	7.5
10 - 11	Ls	2	4 × 3 × $\frac{5}{16}$	8.833	7.1	2	125	2.5
11 - 12	Ls	2	$3\frac{1}{2}$ × 3 × $\frac{5}{16}$	11.5	6.6	2	152	3
12 - 13	Ls	2	$2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{1}{4}$	8.33	4.0	2	67	1.5
13 - 14	Ls	2	$3\frac{1}{2}$ × 3 × $\frac{5}{16}$	9.00	6.6	2	119	2.5
14 - 15	Ls	2	3 × 2 × $\frac{5}{16}$	8.25	5.0	2	83	1.5
15 - 16	Ls	2	5 × 3 × $\frac{5}{16}$	7.6	8.2	2	124	2.5
16 - 17	Ls	2	3 × $2\frac{1}{2}$ × $\frac{5}{16}$	8.16	5.5	2	90	2
	Summa from page 33						7213.	310.
	Total =						14778	2414.5

1	2	3	4	5	6	7	8	9
			Summa from page 3			4	14778	2414.5
17-18	L's	2	$4 \times 3 \times \frac{5}{16}$	6.41	21.1	2	91	2
18-19	L's	2	$3\frac{1}{2} \times 3 \times \frac{5}{16}$	8.25	26.6	2	108	2
19-20	L's	2	$4 \times 3 \times \frac{5}{16}$	5.15	21.1	2	82	1.5
20-21	L's	2	$3\frac{1}{2} \times 3 \times \frac{5}{16}$	8.33	26.6	2	110	2
21-22	L's	2	$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$	5.33	14.5	2	48	1
22-23	L's	2	$3\frac{1}{2} \times 3 \times \frac{5}{16}$	8.58	26.6	2	113	2
23-24	L's	2	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	5.2	14.0	2	42	2
Pin	O	1	5×10	12.5	66.76		67	
Web	Pl	1	$5'0'' \times \frac{3}{8}$	3.5	16.5		261	
Nuts		2					12	
Bearing	Pl	2	$43 \times \frac{5}{8}$	2.7	91.37		493	
"	Pl	2	$25 \times \frac{5}{8}$	2.0	53.12		212	
Pedestal	Pl	2	$24 \times 1\frac{1}{4}$	2.5	102.0		510	
Conn.	L's	2	$6 \times 6 \times \frac{1}{2}$	2.5	19.6		78	
Bed	Pl.	1	$30 \times \frac{3}{4}$	2.5	76.5		191	
Anchor	Bolts	2	$1\frac{1}{4}$	4.0	41.73		33	
Rivets		94	$\frac{7}{8}$	100 @	24.3		23	
Conn.	Rods	1	2×2	142.25	13.6		1940	
Details			2×2	4.5	13.6		61	
Turnbuckles		1					12	
Pin	O	$\frac{1}{2}$	$3 \times 2\frac{3}{4}$	0.438	20.2		4	
Nuts		1					3	
Bearings	Pl.	2	$24 \times \frac{1}{2}$	2.5	40.8		204	
Filler	Pl.	2	$12 \times \frac{1}{2}$	2.4	20.4		96	
			Total				19598	2427
							2427	
							22025	lbs.

Total weight of $\frac{1}{2}$ inch = 22025 lbs.
Number of half arches = 22
Total weight of all arches
= 22 × 22025 = 484550 lbs.

The Skylight Roof

1	2	3	4	5	6	7	8	9
Columns	ls	4	$3\times2\frac{1}{2}\times\frac{1}{4}$	10.0	4.5	33	180	100
26 28 30 } Y	ls	4	$4\times3\frac{1}{2}\times\frac{5}{16}$	24.6	7.6		746	
Low Chord	ls	2	$4\times3\times\frac{5}{16}$	45.0	7.1		639	
25-A	ls	4	$3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	10.3	4.9		202	
26-27	ls	4	$2\times2\times\frac{1}{4}$	2.6	3.19		33	
27-28	ls	4	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	7.0	4.0		112	
28-29	ls	4	$2\times2\times\frac{1}{4}$	5.6	3.19		71	
29-30	ls	4	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}$	9.0	4.0		144	
30-31	ls	1	$3\times2\times\frac{5}{16}$	9.0	5.0		45	
Details of one Truss						5		100
				Totals			2172	160
Total weight of one truss =							160	
							2332	lbs.
Number of trusses = 11								
Total weight of all trusses = 11×2332 =								25652
Conn	Pl.	264	$5\times\frac{3}{8}$	1.0	6.38			1685
"	ls	102	$3\times3\times\frac{1}{4}$	0.698	4.9			350
Purlins	[5	8	10"	270	25.0			54000
Girts	[5	4	1"	270	13.25			17920
Rivets		1200	$\frac{3}{4}$	100 @	16.12			194
Wall	Pl.	16	$4\times\frac{1}{4}$	0.6	34			33
Total weight of skylight roof =								99834 lbs.

Purlins, Rafters etc.

1	2	3	4	5	6	8	9
Window Frame	∠s	208	4 × 3 × 3/8	6.0	8.4	1747.5	
Bracing Rods		72	5/8	26.5	1.043	1990	
Nuts		145				9	
Purlins	⊏	2	10"	2.0	15.0	5100	
"	I	12.	10"	270	25.0	81150	
Connection	Pl.	308	5 × 3/8	1.0	6.38	1963	
Rivets		2150	3/4	100 @	16.1	347	
Bent	Pl	154	24 × 3/8	0864	30.6	4070	
Rivets		930	3/4	100 @	16.12	150	
"		616	3/4	100 @	16.12	99	
Wall	Pl	28	4 × 1/4	06	34	57	
Rafters	⊏	108	4"	7.83	5.25	4440	
"	⊏	108	4"	8.17	5.25	4635	
"	⊏	108	4"	8.67	5.25	4920	
"	⊏	108	4"	9.25	5.25	5250	
"	⊏	108	4"	10.17	5.25	6660	
"	⊏	108	5"	11.75	6.5	8260	
"	⊏	108	5"	15.6	6.5	10960	
Wall Conn.	I	2	10"	270	25	13500	
Connection	∠s	2160	6 × 4 × 3/8	0146	12.3	3880	
"	∠s	864	6 × 4 × 3/8	0208	12.3	2207	
Rivets		3025	3/4	100 @	16.12	487	
Subpurlins	∠s	121	1 1/2 × 1 1/2 × 3/16	270	1.79	58500	
Rivets		22896	1/2"	100 @	4.82	1101	
Bracing Rods		24	5/8	53	1.043	1320	
"		24	3/4	54	1.502	1949	
"		24	1"	59	2.67	3782	
Nuts		228				59	
Gallery Girts	⊏	2	15"	270	40.0	21600	
Knee braces	∠s	80	3 × 3 × 1/4	13.5	4.9	5290	
Connection	Pl	20	24 × 1/4	3.5	20.4	1430	
			Total =			268631	lbs.

1	2	3	4	5	6	8	7
			From page 37			268631	
Connection	L	22	12 × ¼	3.5	12.2	1113	
Rivets		900	¾	100 @	16.12	144	
Shelf	Ls	22	6 × 4 × ½	0.677	16.2	242	
Wall	Pl	4	4 × ¼	0.6	3.4	8	
Connection	Pl	18	24 × ¼	1.08	2.04	396	
Gallery Joists	I	2	15"	270	55	29700	
Knee braces	Ls	80	6 × 4 × ⅜	13.5	12.3	13280	
Connection	Pl	20	24 × ¼	3.5	20.4	1430	
"	Pl	22	12 × ¼	3.5	10.2	785	
Rivets		900	¾	100 @	16.12	144	
Shelf	Ls	22	6 × 4 × ½	0.677	16.2	242	
Wall	Pl	4	4 × ¼	0.6	3.4	8	
Connection	Pl	18	24 × ¼	1.08	20.4	396	
Balcony Joists	I	2	15"	270	33.0	17800	
Knee braces	Ls	80	3 × 3 × ¼	14.5	4.9	5678	
Connection	Pl	20	24 × ¼	3.5	20.4	1430	
"	Pl	22	12 × ¼	3.5	10.2	785	
Rivets		900	¾	100 @	16.12	144	
Shelf	Ls	22	6 × 4 × ½	0.677	16.2	242	
Wall	Pl	4	4 × ¼	3.4	0.6	8	
Balcony Joists	I	2	15"	270	55.0	29700	
Knee braces	Ls	80	6 × 4 × ⅜	12.5	12.3	12300	
Connection	Pl	20	24 × ¼	3.5	20.4	1430	
"	Pl	22	12 × ¼	3.5	10.2	785	
"	Pl	36	24 × ¼	1.08	20.4	792	
Rivets		900	¾	100 @	16.12	144	
Shelf	Ls	22	6 × 4 × ½	0.677	16.2	242	
Wall	Pl	4	4 × ¼	0.6	3.4	8	
Gallery Joists	I	108	10"	180	25.0	48650	
Rail.	Ls	216	2 × 2 × ¼	4.0	3.14	2753	
Floor bars	▭	108	5 × 1	15.0	17.0	27550	
Connection	Ls	216	6 × 4 × ⅜	0.42	12.3	1115	
			Total	=		467,494	Lbs.

1	2	3	4	5	6	8	9
	From page 28					116,447	
Connection	Pl	432	6 × 1/4	1.0	5.1	2200	
Rivets		3620	3/4	100 @	16.12	708	
Balcony joists	I	108	10"	32.08	23.0	86,750	
" "	I	108	6"	12.5	12.25	10895	
" "	I	4	4"	270	1.5	8100	
Bed	Pl.	4	4 × 1/4	270	3.4	3668	
Floor bars	▭	108	5 × 1	53	17.0	97,400	
Connection	∠s	216	6 × 4 × 3/8	0.42	12.3	1115	
"	Pl	216	6 × 1/4	1.0	5.1	1100	
Rivets		8218	3/4 100 @		16.12	1322	
			Total	=		68,207	lbs

Summary.

Weight of the three hinged Arches = 484550
" " " Skylight roof = 79834
" " " Purlins Rafters etc. = 687207
Total weight of steel frame = 1271591

1271591 lbs = 635.795 tons.

Actual Dead Load per sq. foot of Surface:

Weight of gallery not supported by arch = 84300 lbs.

1271591 − 84300 = 1187291 lbs.

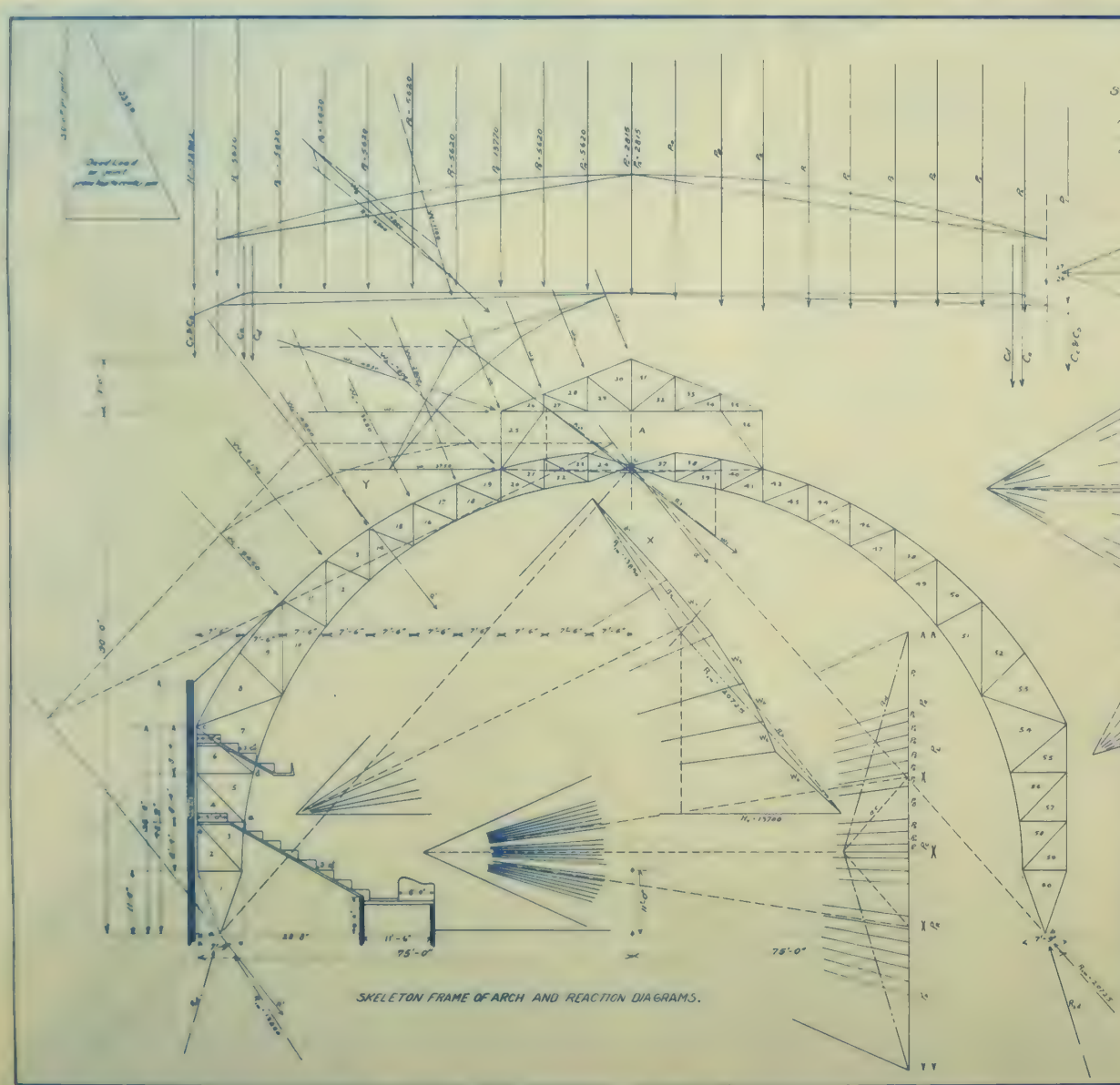
Area of sides of building = 270 × 36 × 2 = 72000 sq. feet
" " horizontal projection of frame = 270 × 150 − 40600
Total area of steel frame = 112600 square feet.

Average load per square foot of surface due to actual dead load = $\frac{1187291}{112600}$ = 10.52 lbs.

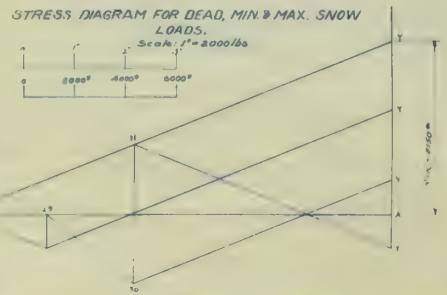
Assumed dead load (page 6.) = 13.0 lbs.

Estimate of Cost of Steel Frame

Material	Cross Section	Weight in lbs.	Cost in Cents pr. lbs Shop Cost.	Mill Cost	Sum	Total Cost in Dollars
Arches		484550	0.75	2.50	3.25	15748
Trusses		25625	0.75	2.40	3.15	808
Purlins	I & L	161170	0.25	2.75	3.00	4835
Connection	L	7552	1.52	2.40	3.95	298
Rafters	I	45125	0.25	2.75	3.00	1354
Subpurlins	L	58500	0.15	2.40	2.55	1492
Window frames	L	10475	0.50	2.40	2.90	304
Rods		9100	1.25	2.45	3.70	337
Joists	I	160395	0.25	2.75	3.00	4812
Girt frame	[& L	136316	0.50	2.65	3.15	4294
Terrace bars		124950	1.50	2.50	4.00	4998
Plates		21471	1.55	2.50	4.05	870
Rail	L	2753	0.55	2.40	2.95	81
Plain material		23582			2.55	601
Total =		1271591 lbs.				
Paint	2 coats 1st coat = $\frac{1}{2}$ gal pr. tons					
	2de " = $\frac{3}{8}$ " " "					
	317.9 + 238.5 = 556.4 @ $1.20 =					668
Labor of painting						1300
Shipment from Pittsburg to Peoria						
	a 0.2 cents a pound					2543
Erection - $10.00 pur ton						6358
Drafting @ $2.00 pr tons						1272
Total net cost of steel frame = $						52973.00



SKELETON FRAME OF ARCH AND REACTION DIAGRAMS.



STRESS DIAGRAM FOR DEAD, MIN. & MAX. SNOW LOADS.
Scale: 1" = 2000 lbs

COLISEUM AND EXPOSITION BUILDING.

For

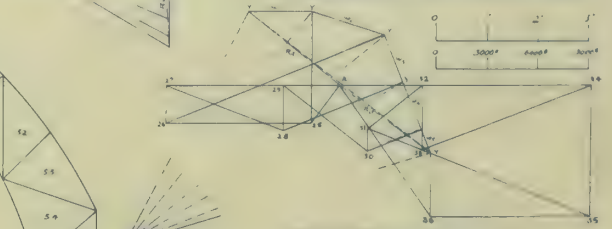
PEORIA. ILL.

DESIGNED BY

C.P.L. PETERSEN.

UNIVERSITY OF ILLINOIS. 1903.

REACTION DIAGRAMS, AND STRESS DIAGRAMS FOR SKYLIGHT ROOF.



STRESS DIAGRAM FOR WIND LOADS.
Scale 1" = 3000 lbs

Stress Sheet

Member	Dead Loads	Max Snow	Min Snow	Wind Loads	Wind + Dead + Min Snow	Max Snow + Dead Load	Wind + Dead Load
R1	8150			6900			
R2	8150			6700			
Y 25	-8150	+11230	-5615	+4700	+30200	+3080	+14850
A 25	0	0	0	-2400	-2400	0	-2400
25 26	-17000	-23220	-11610	-14400	-41620	-40300	-31400
26 Y	-18300	+23210	+12105	-12600	-43300	-43300	-30900
26-27	0	0	0	-3200	-3200	0	-3200
A 27	-17000	-23220	-11610	-9300	-36120	-40300	-26300
27-28	-3700	+5110	+2550	-2400	-8100	+1400	-6100
Y 29	-18800	+20100	+10050	-7600	-33250	+1300	-26450
28 27	-1300	1790	895	-1200	-3395	490	-2500
A 29	13500	18600	-9300	+4500	-18300	32100	-4800
29 30	-4900	+6100	+3050	-1200	-7150	+10900	-6100
Y-30	+10800	+15100	+7550	+4000	+33450	+25900	+18900
30-31	-5400	-7450	-3728	-1600	-10500	-12300	-6900

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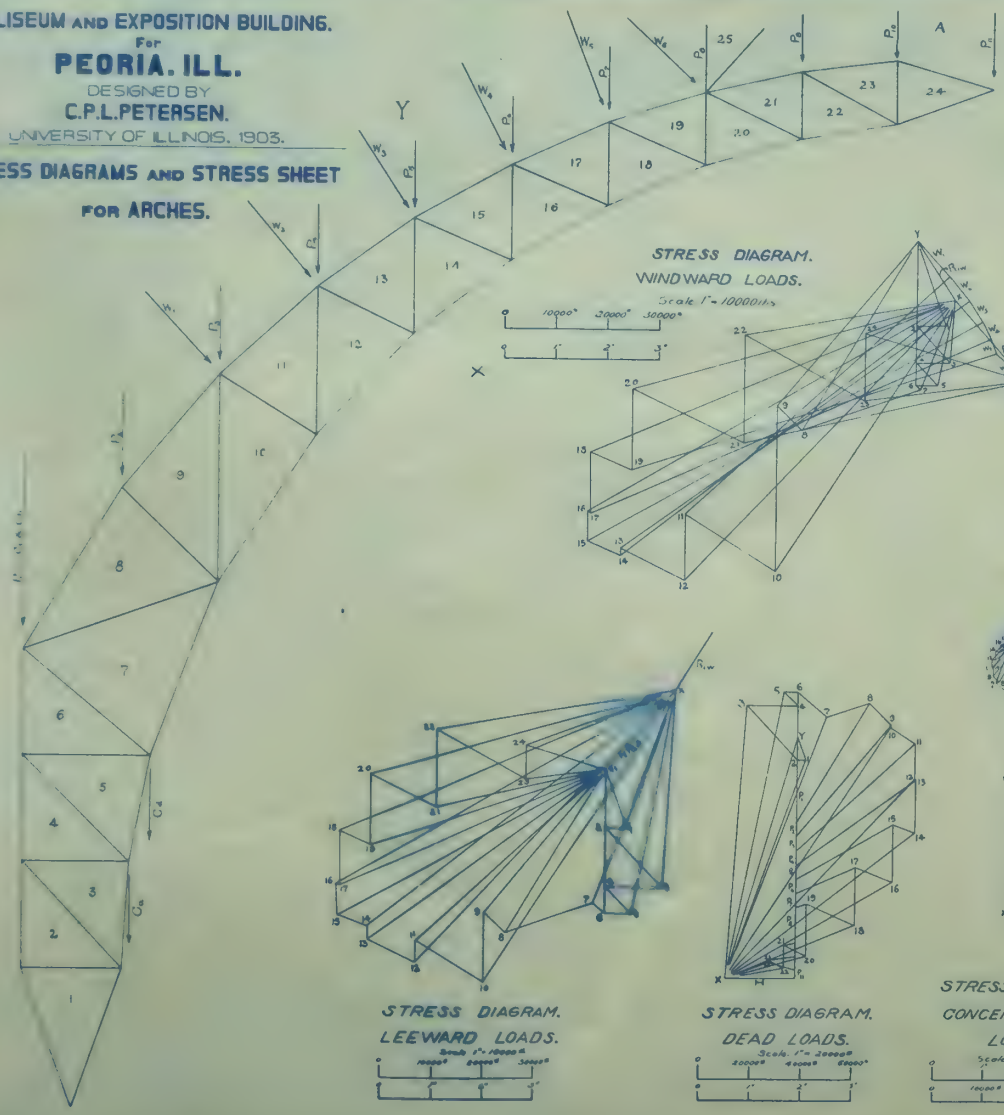
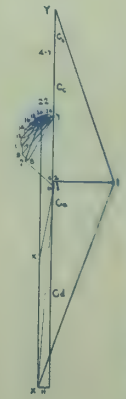
STRESS DIAGRAMS AND STRESS SHEET FOR ARCHES.



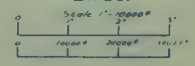
STRESS DIAGRAM.
WINDWARD LOADS.
Scale 1"=10000lbs

STRESS DIAGRAM.
LEEWARD LOADS.

STRESS DIAGRAM.
DEAD LOADS.
Scale 1"=20000#

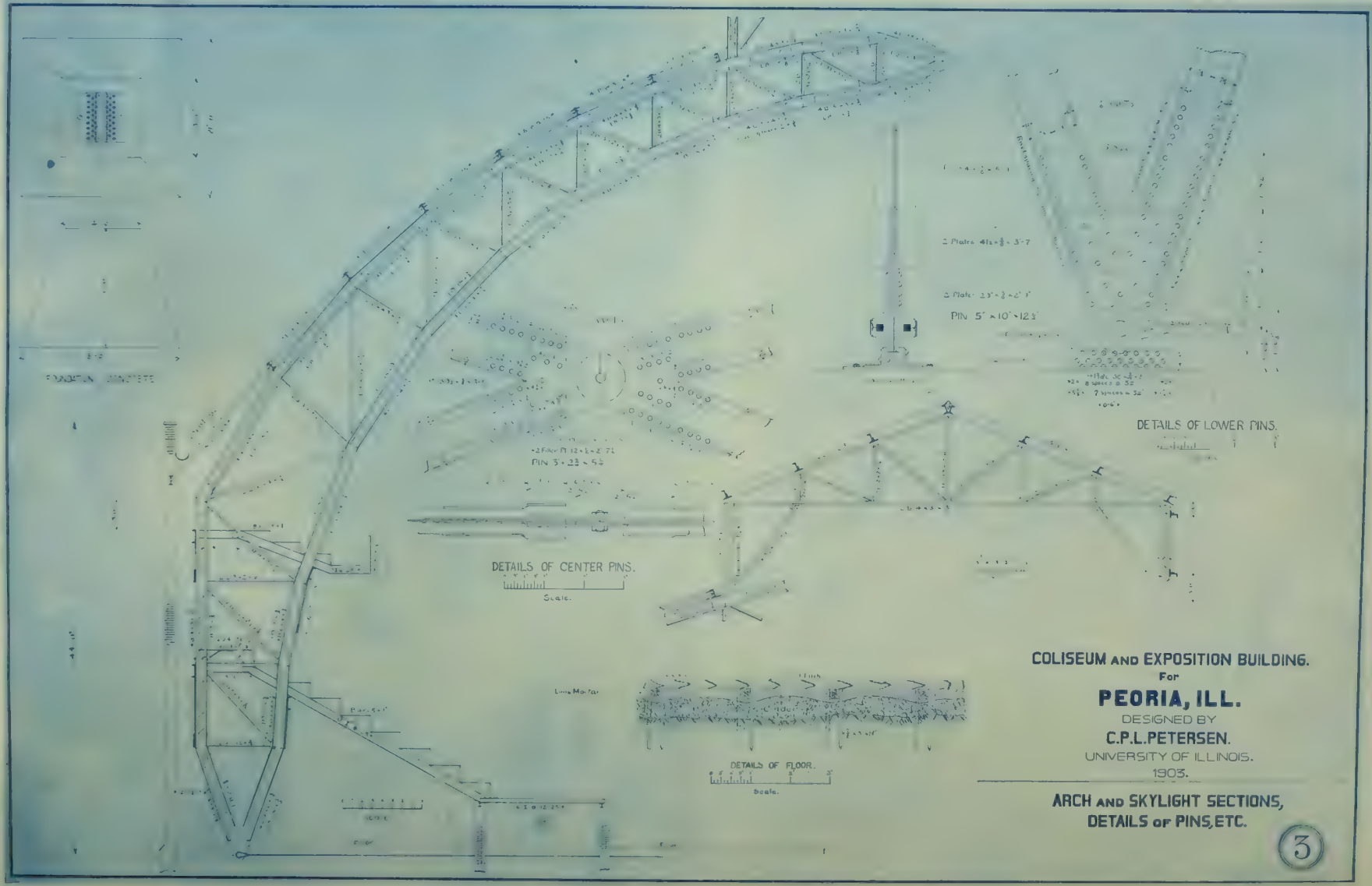


STRESS DIAGRM.
CONCENTRATED
LOADS.
Scale 1"=10000#



STRESS SHEET.

Members	Dead Load.	Max. Snow Load.	Min Snow Load	Concentrated Load	Wind Load	Wind + Concentrated Load + Min Snow	Max Snow + Concentrated Load + Dead Loads	Wind + Dead Load
R_1	39060	59300	29650	75000	13850	VC 211600	VC 226400	VC 116400
R_2	99000	59300	29650	75000	20725	VC	VC	VC
1-Y	+ 9500	+ 5700	+ 2850	+ 36000	+ 17600 - 18000	+ 55350	+ 51200	+ 27100 - 18000
1-X	+ 91300	+ 54500	+ 27250	+ 43400	- 5400 + 28400	+ 130350	+ 189200	+ 119700 - 5400
1-2	- 3500	- 2090	- 1050	- 12000	- 5800 - 4100	- 22350	- 17590	- 4100 - 3700
2-Y	+ 19000	+ 5400	+ 2700	+ 34000	+ 18500 - 11200	+ 62200	+ 48400	+ 39500 - 11200
2-3	- 29100	- 17400	- 8700	+ 500	+ 9800 - 16100	- 53900	- 46500	+ 9800 - 45500
3-X	+ 107800	+ 64000	+ 32000	+ 40300	- 13200 + 38800	+ 218900	+ 214100	+ 146100 - 12800
3-4	+ 20000	+ 11900	+ 5950	- 600	- 6700 + 11100	+ 37050	+ 31900	+ 31100 - 6700
4-Y	- 18800	- 7700	- 3850	+ 26000	+ 23900 - 23000	- 39650	- 20500	+ 23900 - 23000
4-5	- 7800	- 4650	- 2325	+ 500	+ 6700 - 7200	- 17325	- 12450	+ 6700 - 7200
5-X	+ 115000	+ 68700	+ 34350	+ 14600	- 17000 + 44800	+ 208750	+ 198300	+ 130600 - 17000
5-6	+ 5700	+ 3400	+ 1700	- 200	- 4400 + 5100	+ 13500 - 4600	+ 9100	+ 10000 - 4400
6-Y	- 18000	- 10700	- 5350	+ 26500	+ 28200 - 28200	- 31350 + 34700	- 28700 + 28500	+ 28200 - 28200
6-7	+ 14900	+ 9950	+ 4980	- 7200	+ 1200 + 3000	+ 22880	+ 24850	+ 17900 - 7200
7-X	+ 110000	+ 73500	+ 36800	- 9400	- 18800 + 44900	+ 191700	+ 183500	+ 130100 - 18800
7-8	+ 18000	+ 12000	+ 6000	+ 500	- 24600 + 18100	+ 42600	+ 3050	+ 36700 - 24600
8-Y	- 55000	- 36800	- 18400	+ 10000	+ 45500 - 32500	- 110300	- 91800	+ 45500 - 92500
8-9	+ 12000	+ 8000	+ 4000	- 1600	- 6800 + 5800	+ 21800	+ 20000	+ 17400 - 6800
9-Y	- 57100	- 38000	- 19000	+ 9700	+ 42300 - 36700	- 112800	- 35100	+ 43300 - 33800
9-10	+ 1000	+ 670	+ 385	- 1500	+ 32500 - 13800	+ 33885	+ 1670 - 1500	+ 33300 - 13800
10-X	+ 118700	+ 78700	+ 39350	- 7000	- 63900 + 60400	+ 225750	+ 196700	+ 186400 - 63900
10-11	+ 11200	+ 7500	+ 3750	- 250	+ 37000 + 15500	+ 36450 - 21250	+ 18700	+ 36700 - 24000
11-Y	- 62800	- 42000	- 21000	+ 8700	+ 69300 - 50000	- 133800	- 104800	+ 69300 - 117400
11-12	+ 14400	+ 9600	- 4800	- 1700	+ 13000 - 4500	+ 27450 - 17000	+ 24000	+ 27400 - 13500
12-X	+ 108000	+ 72100	+ 36050	- 5900	- 76200 + 74800	+ 217850	+ 180100	+ 181300 - 76200
12-13	+ 500	+ 330	+ 160	+ 600	- 14000 + 10400	+ 11660 - 14000	+ 1430	+ 18900 - 14000
13-Y	- 57800	- 38500	- 19250	+ 7000	+ 31300 - 52100	- 130750 + 36800	- 96300	+ 83800 - 118300
13-14	+ 20900	+ 14000	+ 7000	- 800	+ 400 + 2200	+ 30600	+ 34900	+ 25600 - 6000
14-X	+ 94000	+ 62800	+ 31300	- 4200	- 82500 + 73900	+ 201200	+ 156600	+ 169300 - 82500
14-15	- 9100	- 6600	- 3300	+ 800	- 7000 + 6000	- 19400	- 15700	+ 6000 - 16100
15-Y	- 43800	- 29300	- 14650	+ 3600	+ 88400 - 59400	- 117850 + 34000	- 73100	+ 88400 - 103200
15-16	+ 22300	+ 15300	+ 7650	- 100	- 6000 + 6100	+ 36650	+ 38200	+ 23000 - 6000
16-X	+ 75700	+ 50500	+ 25300	- 3000	- 83000 + 76000	+ 177050	+ 126400	+ 149800 - 83000
16-17	- 15600	- 10400	- 5200	+ 1100	+ 900 - 300	- 21700	- 26000	+ 900 - 6500
17-Y	- 25600	- 17100	- 8550	+ 4000	+ 85500 - 56600	- 90750 + 89500	- 42700	+ 85500 - 62500
17-18	+ 23000	+ 15300	+ 7650	- 1000	- 12100 + 16600	+ 41450	+ 38300	+ 23100 - 12100
18-X	+ 55000	+ 36700	+ 18350	- 500	- 77100 + 71000	+ 144350	+ 91700	+ 126700 - 77100
18-19	- 20100	- 13700	- 6850	+ 1100	+ 9000 - 6800	- 34250	- 34300	+ 9000 - 37400
19-Y	- 4500	- 3000	- 1500	+ 2300	+ 24800 - 18100	- 34100 + 27200	- 7500	+ 24800 - 18100
19-20	+ 20500	+ 13700	+ 6850	- 800	- 15800 + 14000	+ 41350	+ 34200	+ 34900 - 15800
20-X	+ 33000	+ 22000	+ 11000	- 200	- 65800 + 61500	+ 105500	+ 55000	+ 63900 - 65800
20-21	- 9600	- 6400	- 3200	+ 300	+ 34900 - 27000	- 42200	- 16000	+ 34900 - 27000
21-A	+ 4600	+ 310	+ 155	+ 2000	+ 56300 - 54000	+ 64450	+ 9700	+ 10300 - 14000
21-22	+ 9200	+ 6100	+ 3050	- 600	- 47000 + 42000	+ 27150	+ 15300	+ 10300 - 47000
22-X	+ 23300	+ 15500	+ 7750	+ 500	- 41700 + 42000	+ 78350	+ 39300	+ 40300 - 41700
22-23	- 7000	- 4700	- 2350	+ 600	+ 26700 - 20000	- 29350 + 27300	- 11700	+ 26700 - 27000
23-A	+ 11000	+ 7400	+ 3700	+ 1200	+ 32000 - 15700	+ 47900	+ 19600	+ 43300 - 15700
23-24	+ 700	+ 470	+ 235	- 600	- 15200 + 16000	+ 7235 - 13800	+ 1170	+ 7300 - 15200
24-A	+ 11500	+ 7700	+ 3850	+ 1200	+ 32800 - 16700	+ 49350	+ 20400	+ 44100 - 16700
24-X	+ 18000	+ 12000	+ 6000	+ 1200	- 18000 + 31000	+ 56200	+ 31200	+ 49200 - 18000
H	28000	18700	9350	2300	7350 27500	39650	43000	43500



COLISEUM AND EXPOSITION BUILDING.

For

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1903.

ARCH AND SKYLIGHT SECTIONS,
DETAILS OF PINS, ETC.

3

COLISEUM AND EXPOSITION BUILDING.

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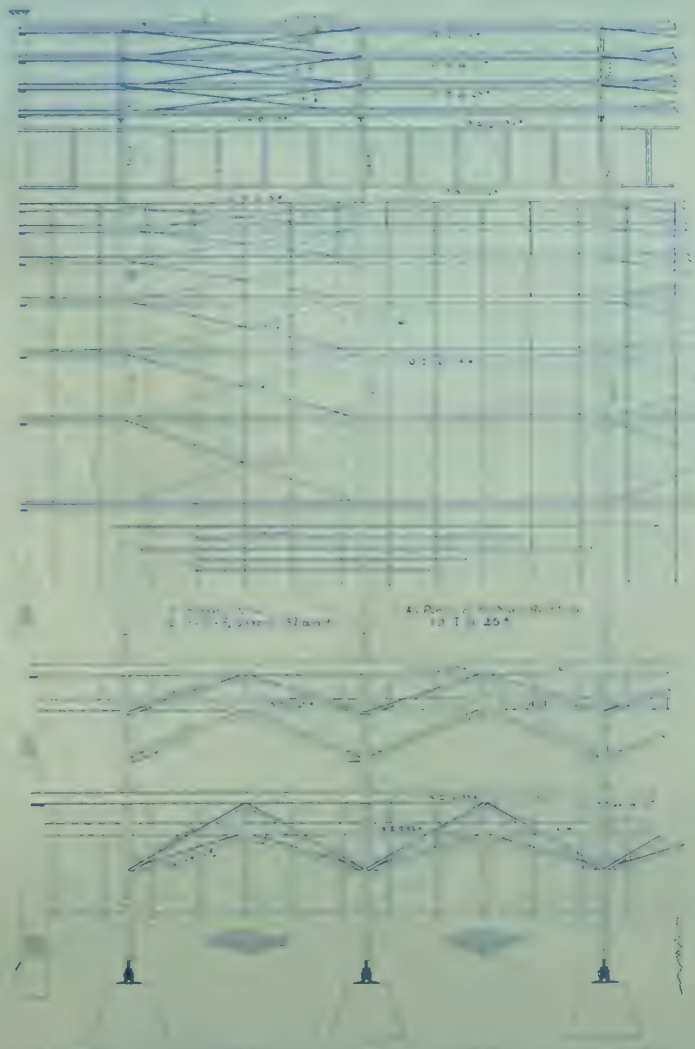
1903.

SIDE ELEVATION.

Ludowici Interlocking Tiles (T. I. - 3"+12")
Catalogue No 4 1902.
The Roof of Towers and over the Entrance
will be covered as the Main Roof



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1903.

FACADE AND FLOOR PLAN.

5

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